Evaluation of Existing CPT Correlations in Silt

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ABSTRACT: This paper evaluates the applicability of existing Cone Penetration Test (CPT) soil type and soil properties correlations in two different deposits of uniformly graded non-plastic silt. Current CPT correlations for soil engineering properties are based largely on experience in either sands that are typically drained during penetration, or clays that are typically undrained. Silts may exhibit partially drained conditions during penetration that introduces uncertainty when applying correlations from the literature. The assessment is based on an analysis of existing CPT data collected at two study sites in Rhode Island, U.S.A. that are underlain by thick deposits of non-plastic silt. Existing CPT correlations were used to predict the soil type and selected geotechnical properties of the silt, which were compared to laboratory test results to evaluate the quality of the predictions. The silts in this study exhibited partially drained to drained behaviour during cone penetration. Existing CPT soil classification charts were ineffective in identifying the silt but correctly characterized its engineering behaviours. Existing CPT correlations accurately predicted the friction angle, shear wave velocity, and cyclic resistance of the silts investigated in this study.

1. INTRODUCTION

Silt deposits are encountered worldwide and are the dominant soil deposit in coastal urban areas of Rhode Island, U.S.A. The focus of this study is on uniformly graded non-plastic silt classified as "ML - Silt" in accordance with the Unified Soil Classification System (USCS). Under drained conditions the mechanical behaviours of silt is most similar to sand. However, their relatively low permeability makes them particularly susceptible to pore pressure generation that has an adverse affect on geotechnical performance including softened axial pile response (Bradshaw et al. 2012), ground movements during construction (Bradshaw et al. 2007a), and liquefaction during cyclic loading (Bradshaw et al. 2007b; Baxter et al. 2008).

Silts are extremely susceptible to disturbance during sampling, handling, and testing, and thus in-situ tests such as the Cone Penetration Test (CPT) are important tools for estimating soil properties for design. Often these soil properties are estimated from empirical correlations derived from case studies. However, since the majority of existing correlations were developed in sands or clays, the accuracy of these correlations in silts is somewhat uncertain.

The objective of this study was to validate a number of existing CPT correlations in uniformly graded silt. The study uses existing CPT data collected at two silt sites in Rhode Island. Laboratory test data on the same soils are used to benchmark the results obtained using the CPT correlations. Sample disturbance in the laboratory testing program is taken into account by preparing samples to given values of shear wave velocity that match in situ values (Bradshaw and Baxter 2007; Baxter et al. 2008). This paper will describe the study sites, evaluate drainage conditions during cone penetration, and investigate a number of predictive correlations used for soil classification, effective friction angle, shear wave velocity, and cyclic resistance ratio.

2. STUDY SITES

2.1 CPT Data

CPT data was obtained at two silt sites in Rhode Island adjacent to Narragansett Bay. The silts were deposited as glacial lake sediments during the last glacial retreat and thus are Holocene in age. The silts typically contain seasonal varves with coarser material being deposited in summer when erosion is most active and finer material deposited in winter. The Standard Penetration Test (SPT) continues to be the standard of practice in the region, and it is interesting to note that these are the only two sites in the state where CPT has ever been utilized in a site investigation program.

The first site called *Wellington Avenue* is the site of a former railway expansion project located about 5 km south of the city of Providence. CPTs were performed by Geocomp Corporation using a standard cone having a 10 cm^2 cone tip area with pore pressure measured at the cone shoulder (u_2 position) and pushed with a conventional drill rig (DMJM Harris 2001). A representative CPT profile of the soil conditions at Wellington Avenue including cone tip resistance corrected for unequal area effects (q_t), sleeve friction (f_s), and pore water pressure (u) is shown in Figure 1. The q_t traces from nine other CPTs that were performed at the site are also shown in the figure and illustrate the high lateral variability across the site. Some of the variability is due to the irregular topography at the site when the results are plotted with depth. Pre-boring was performed through the fill layer that had variable thickness across this site prior to performing penetration testing. Conventional borings were also performed adjacent to the CPT locations. Shelby tube samples of silt were recovered during drilling and laboratory tests were conducted on selected samples to determine index properties.

The second site is called the *Farmers' Market* and located in downtown Providence. The authors performed seismic cone testing at the site as part of a study on the liquefaction potential of Providence silt (Bradshaw 2006; Bradshaw et al. 2007b; Baxter et al. 2008). Three seismic CPTs were performed by ConeTec using a standard seismic cone with a 15 cm^2 cone area and pore pressure measurement in the u_2 position and pushed with a 20 ton cone rig. Shear wave travel time measurements were made "down-hole" and values of shear wave velocity were measured at 1-meter depth intervals. A representative CPT profile at the Farmers' Market is shown in Figure 2. Again pre-boring was performed through the upper fill layers before the cone was pushed. The q_t traces from two other cone tests at the site are shown in Figure 2 and indicate the very low lateral variability across the site. Conventional borings were also performed primarily to collect representative samples of silt using a large diameter (7.5 cm) split-spoon sampler with a core catcher. Fixed piston sampling was attempted but was unsuccessful.

The profiles in Figures 1 and 2 show cone tip resistances on the order of 5 MPa. Both profiles indicate that excess pore water pressures were generated during penetration. A review of all CPT profiles indicated that both positive and negative excess pore pressures were generated during penetration. The magnitude of the cone tip resistance seemed to reflect the pore pressure response. For example, as shown in Figure 2, the cone tip resistance is lowest at depths where positive excess pore water pressures were highest (i.e. from 16 to 20 m and 30 to 33 m depth).

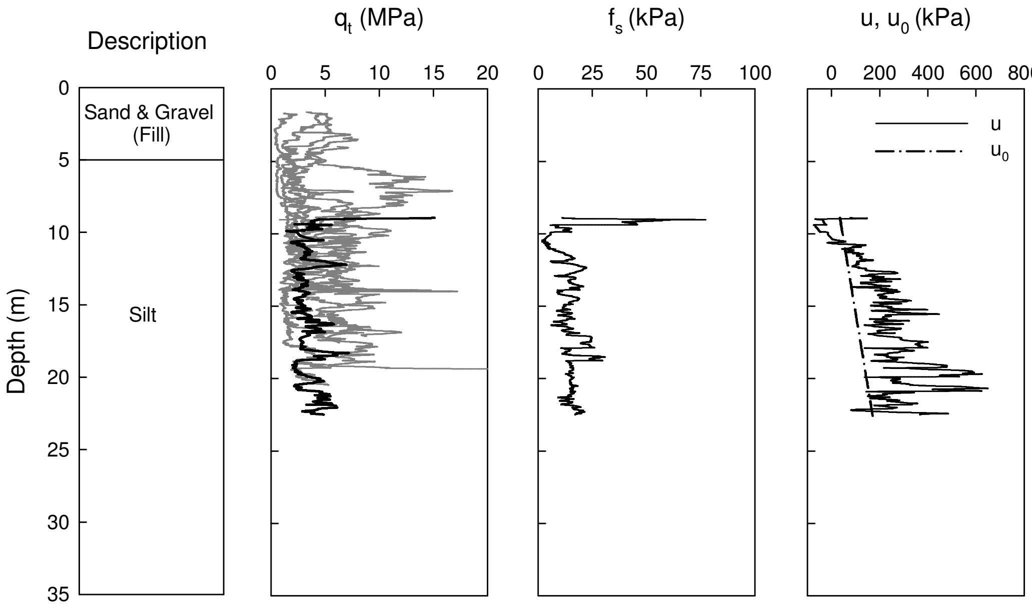


Figure1 Typical CPT profile (CPT-21) from the Wellington Avenue site. The q_t traces from nine other CPTs from the site are shown in gray

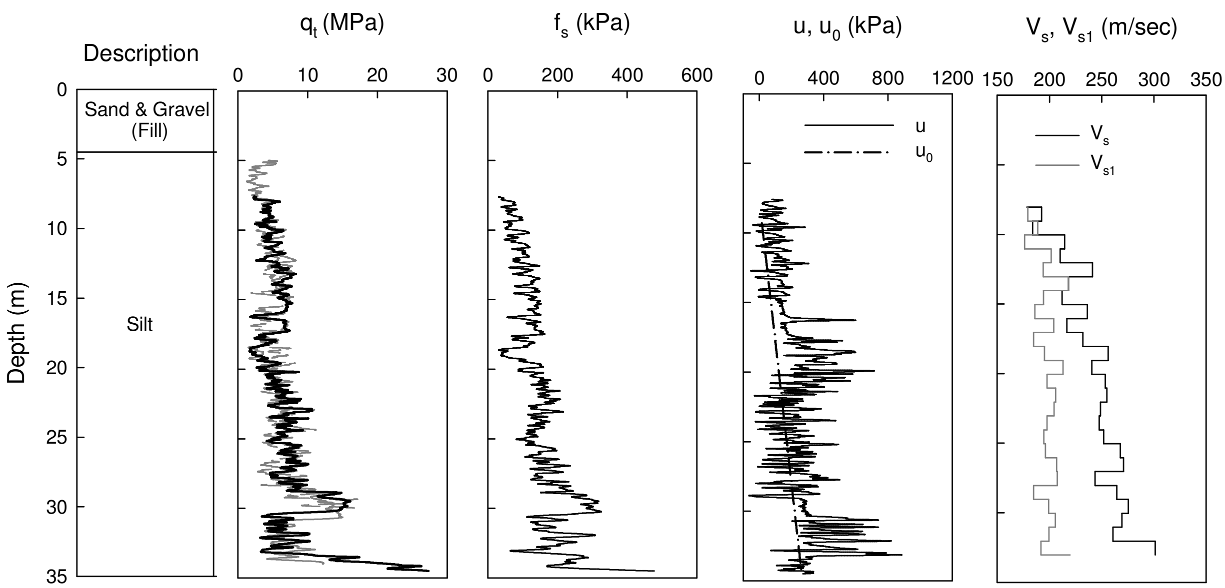


Figure 2 Typical CPT profile (CPT-1) from the Farmers' Market site. The q_t traces from two other CPTs from the site are shown in gray

2.2 Laboratory Data

Laboratory tests performed on silt samples recovered from the study sites were used as a baseline for assessing various CPT soil properties correlations discussed herein. Grain size analyses and Atterberg limits performed on selected samples at the Wellington Avenue site were used as a basis for evaluating soil classification charts. The index testing was performed by Geocomp Corporation and the results are summarized in a geotechnical report (DMJM Harris 2001). Specific gravity and grain size analyses were also performed on the samples used in the laboratory testing described next and the results are summarised in Table 1.

Table 1 Summary of index properties of silt tested in this study

Soil	G_s	% < 0.074 (mm)	% < 0.005 (mm)	D_{50} (mm)
Farmer's Market	2.70	95	6	0.033
Wellington Avenue	2.78	99	9	0.018

2.2.1 Sample Preparation

Triaxial tests on reconstituted samples of silt were performed to assess both monotonic and cyclic strength. The challenge to preparing reconstituted samples is achieving the same void ratio and fabric condition as the soil in situ. However, previous studies by the authors comparing test results from a block sample demonstrated that the field conditions could be replicated in the laboratory by using a modified moist tamping procedure to reconstitute test specimens (Bradshaw and Baxter 2007) in combination with shear wave velocity measurements (Baxter et al. 2008).

The Modified Moist Tamping (MMT) procedure involves tamping the soil at a specified hammer drop height (i.e. tamping energy) and specified molding water content (or molding saturation, S). This allowed the specimens to be prepared to different “fabrics”. For example, Figure 3 shows the relationship between cyclic stress ratio and number of cycles to reach initial liquefaction for different samples prepared to the same void ratio. The figure shows that when the samples were tamped drier (S=15%) they had a higher cyclic strength than the samples prepared wetter (S=55%). This was because it required more energy in the drier sample to achieve the same void ratio and thus the fabric was stronger. Moreover, it was found that when reconstituted samples were prepared at a molding saturation of 55% than the cyclic strength matched the strengths of specimens trimmed from an undisturbed block sample.

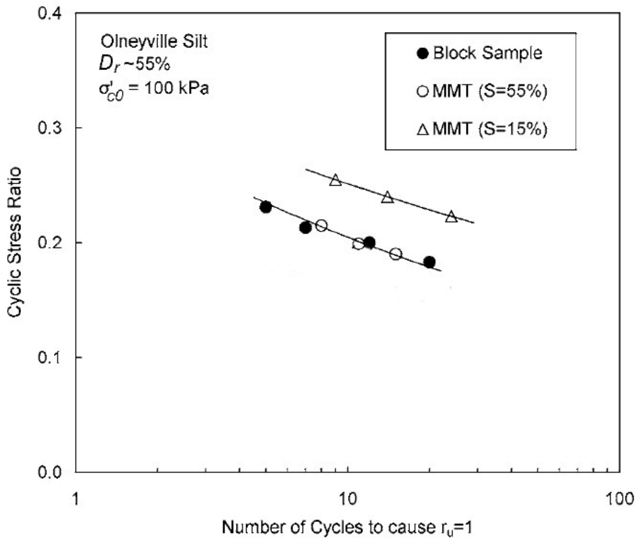


Figure 3 Comparison of cyclic triaxial results from samples prepared by modified moist tamping and from an undisturbed block sample. Note that the S refers to the molding saturation (modified from Bradshaw and Baxter 2007).

2.2.2 Cyclic Triaxial Tests with Shear Wave Velocity Measurements

The cyclic resistance of the Farmers' Market silt was assessed using correlations that were developed from cyclic triaxial tests on site-specific soils that relate Cyclic Resistance Ratio (CRR) to overburden stress-corrected shear wave velocity (V_{s1}). CRR is defined as the ratio of cyclic shear stress to the initial vertical effective stress. In the field V_{s1} is typically calculated from the following equation:

$$V_{s1} = V_s \left(\frac{P_a}{\sigma_{v0}'} \right)^{0.25} \tag{1}$$

where V_s=measured shear wave velocity, P_a=reference pressure (~100 kPa), and σ'_{v0}= initial effective overburden stress.

Cyclic resistance and shear wave velocity measurements were made in isotropically consolidated cyclic triaxial tests, and therefore the results were adjusted to field conditions to compare to the CPT-based CRR predictions presented later on. First, the cyclic resistance results were corrected to direct simple shear conditions and multidirectional shaking by factors of 0.73 and 0.90, respectively. Cyclic resistance was also determined at 15 loading cycles, which is considered to be equivalent to a moment magnitude (M_W) earthquake of 7.5 in existing simplified procedures.

Shear wave velocity was measured in the triaxial specimens using bender elements (e.g., Dyvik and Madshus 1985). The shear wave velocities were corrected for at-rest conditions and stress corrected to a reference stress of 100 kPa by the following equation (Baxter et al. 2008):

$$V_{S1} = V_S K_0^{0.125} \left(\frac{P_a}{\sigma_{c0}'} \right)^{0.25} \tag{2}$$

where V_s=measured shear wave velocity, K_0=lateral earth pressure coefficient at-rest, P_a=reference pressure (~100 kPa), and σ'_{c0}= initial effective isotropic confining pressure.

Baxter et al. (2008) showed that the relationship between CRR and V_{s1} for Providence silt was soil-specific. Moreover, the relationship for any given silt was shown to be independent of the method of sample preparation (i.e. soil fabric). Therefore, a soil-specific CRR-V_{s1} relationship was developed for the Farmers' Market site using representative silt samples recovered from the geotechnical borings at the site. Triaxial test specimens were prepared using MMT to a molding saturation of 55% that is believed to represent the in situ void ratio and fabric as discussed previously.

The resulting CRR-V_{s1} correlation developed for the Farmers' Market silt is shown in Figure 4 and is based on tests performed at initial effective confining pressures of 100 kPa and 200 kPa. The data suggest that the correlation is not influenced by initial confining pressure as is observed in SPT and CPT-based CRR correlations. Therefore, the correlation does not support the use of an overburden stress correction factor (K_σ) for CRR that is recommended in the simplified procedures (Youd et al. 2001). The effects of confining pressure appear to be accounted for in the shear wave velocity normalization process.

The data in Figure 4 was used to fit a polynomial having the following form (Andrus and Stokoe 1997):

$$CRR = a\left(\frac{V_{s1}}{100}\right)^2 + b\left(\frac{1}{V_{s1}^* - V_{s1}} - \frac{1}{V_{s1}^*}\right) \tag{3}$$

where a,b=curve fitting parameters, V_{s1}=overburden stress corrected shear wave velocity, and V_{s1}*=limiting value of V_{s1}. Consistent with Andrus and Stokoe (1997) for clean sands a V_{s1}* of 215 m/s was assumed and regression analysis yielded the following parameters for the Farmers' Market silt: a=0.022 and b=2.00. The best-fit curve is shown as a dashed line in Figure 4.

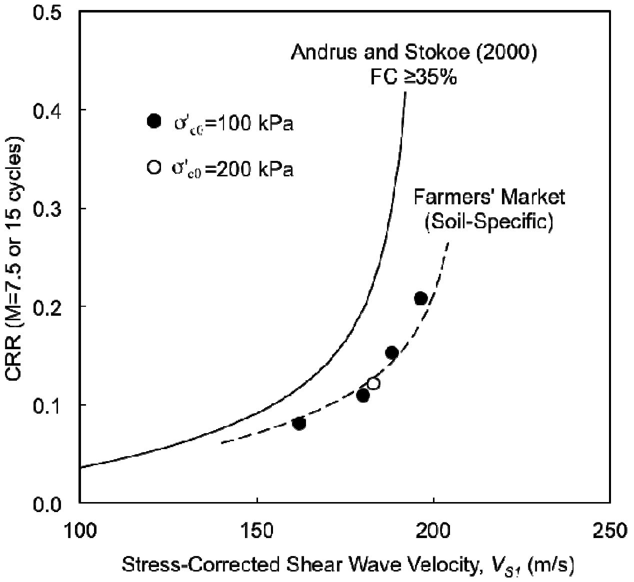


Figure 4 CRR-V_{S1} correlation developed for the Farmers' Market silt. The curve recommended by Andrus and Stokoe (2000) for fines content (FC) above 35% is also shown for reference

2.2.3 Consolidated Drained Triaxial Compression Tests

Isotropically consolidated, drained triaxial compression tests were performed to measure the effective stress friction angle of the Farmers' Market silt. Samples were consolidated to effective confining pressures ranging from 50 to 200 kPa and sheared at a strain rate of 0.01% per minute to ensure drained conditions. V_{s1} calculated using Equation 1 from the seismic cone measurements at the Farmers' Market site showed values ranging from roughly 160 m/s to 200 m/s (Figure 2). Based on data from samples reconstituted to a molding saturation of 55% this corresponds to void ratios ranging from 0.69 to 0.56. Therefore, test specimens were prepared using the modified moist tamping method at a molding saturation of 55% to determine upper and lower bound friction angles for the soil at this site.

3. DRAINAGE CONDITIONS DURING PENETRATION

It is generally accepted that the standard cone penetration rate of 2 cm/s penetration in sands is a drained process and in clays it is an undrained process. However, partial drainage can also occur during penetration in silt (Campanella et al. 1983). Therefore, a good first step when analysing penetration test data in silt is to identify the drainage conditions during penetration.

The excess pore water pressures that are generated during cone penetration is a balance between the pore pressures generated during penetration which is linked to penetration rate and the rate of pore pressure dissipation which is linked to the permeability of the soil. Finnie and Randolph (1994) proposed a dimensionless penetration rate that captures the major factors:

$$V = v d_c / c_h \tag{4}$$

where V=dimensionless penetration rate, v=cone penetration rate, d_c=cone diameter, c_h=coefficient of consolidation for lateral drainage. According to a series of researchers (e.g. Finnie and Randolph, 1994, Chung et al., 2006, Kim et al., 2008) the transition from fully undrained to partially drained conditions occurs when $V \approx 10$. This means that for a CPT carried out using a standard 10 cm^2 cone at a standard rate of 2 cm/s, undrained penetration can be expected in soils with $c_h < 7\text{x}10^{-5}$ m^2/s.

One simple method to determine if CPT penetration occurs undrained or partially drained is to perform a dissipation test. This is where the cone is advanced and stopped and the pore pressures are monitored over time. The transition from undrained to partial drainage would occur at a t_{50} of 0.5 minutes (Robertson et al 1992). Whereas, drained conditions would indicate an almost instantaneous dissipation of excess pore water pressures.

The CPT data from Wellington Avenue included 20 cone dissipation tests. The dissipation test results are shown in Figure 5 as normalized excess pore water pressure plotted against the logarithm of time. As shown in the figure the excess pore water pressures dissipated completely within about 8 minutes with t_{50} ranging from 0.05 to 2 minutes. Based on the criteria above, 4 tests showed undrained behavior, and the remaining 16 tests suggest partially drained to drained behavior. This is consistent with the excess pore pressures that were generated during penetration (Figure 1). Although no dissipation tests were performed at the Farmers' Market site, based on the excess pore pressures shown in Figure 2, it is anticipated that the soil was drained to partially drained. Therefore, the silts are generating excess pore pressures that could have an influence on the measured penetration resistance as compared to drained conditions. The influence of excess pore water pressures on the determination of soil properties through empirical correlations will be explored in the next section.

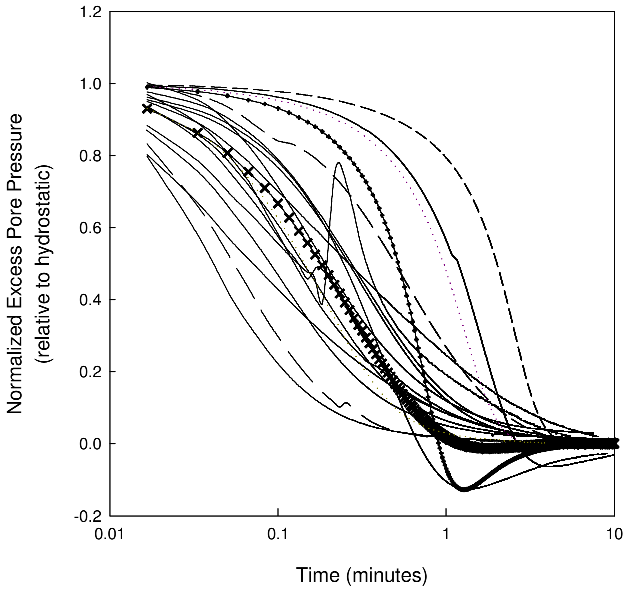


Figure 5 Cone dissipation test curves measured at the Wellington Avenue site

4. ASSESSMENT OF CPT CORRELATIONS

4.1 Soil Classification

Identification of soil type is important for proper characterization of soil mechanical behavior. CPT-based soil classification systems have been developed by numerous researchers including Schmertmann (1978), Douglas and Olsen (1981), Robertson (1990), Jefferies and Davies (1991), Fellenius and Eslami (2000), and Schneider et al. (2008). Robertson et al. (1986) and Robertson (1990) stressed that the CPT classification charts are predictive of Soil Behavior Type (SBT), since the cone responds to the in-situ mechanical behavior of the soil and not directly to soil classification criteria based on grain size distribution and soil plasticity as used in the USCS (Robertson 2010). The primary concern is misidentifying silt as a cohesive (i.e. clay-like) soil.

Robertson (2009) indicates that normalized SBT charts such as those developed by Robertson (1990) and Jefferies and Davies (1991) provide more reliable identification than the non-normalized charts, although when the in-situ vertical effective stress is between

50 kPa to 100 kPa there is often little difference between normalized and non-normalized SBT. The advantage of non-normalized SBT charts is that they can be used during and immediately after the CPT to evaluate the soil type. Normalized SBT charts can only be applied after CPT data has been processed given that it is necessary to know information on soil unit weight and ground water conditions.

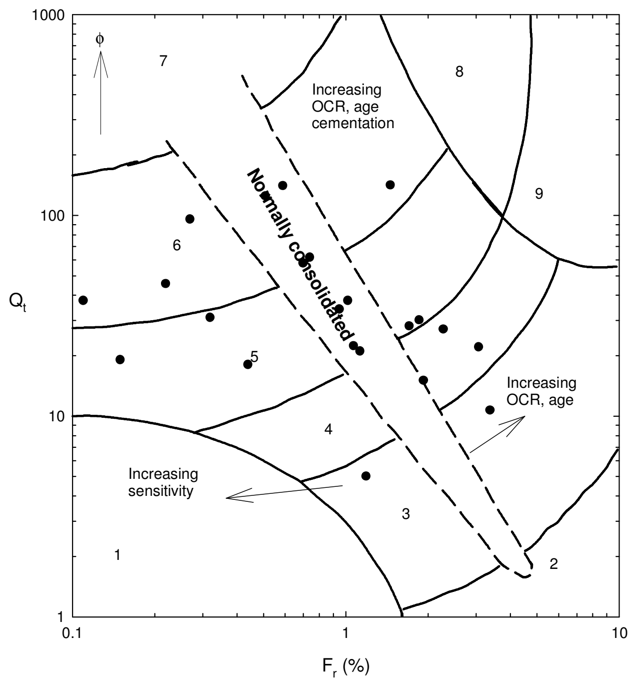
The results of 22 laboratory grain size analysis and Atterberg limits from the Wellington Avenue site were compared to the CPT soil type predictions at the same depths. The CPT data are plotted on four classification charts including Robertson (1990), Jefferies and Davies (1991), Fellinius and Eslami (2000), and Schneider et al. (2008) as shown in Figures 6 through 9. In some of the charts normalized cone values are used as defined below:

$$Q_t = \frac{q_t - \sigma_{v0}}{\sigma_{v0}'} \tag{5a}$$

$$F_r = \frac{f_s}{q_t - \sigma_{v0}} \times 100\% \tag{5b}$$

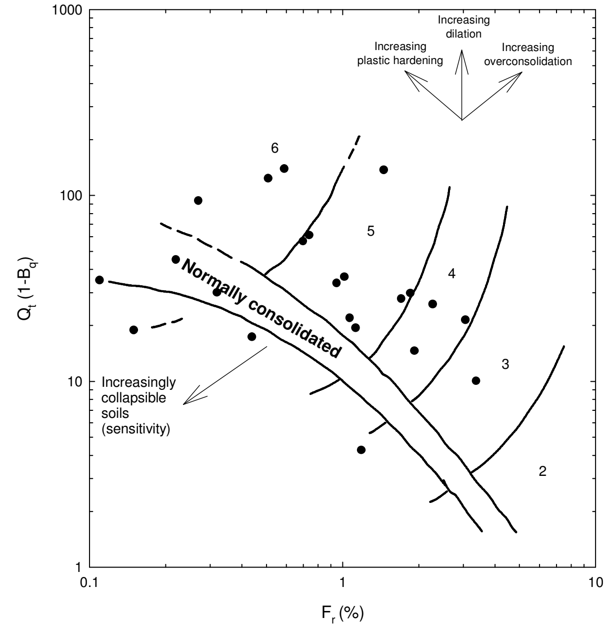
$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{v0}} \tag{5c}$$

where σ_{v0}=initial total vertical stress, σ_{v0}'=initial vertical effective stress, u_2=pore pressure measured at the cone shoulder, and u_0=hydrostatic pressure. The cone stress q_E in Figure 8 is the "effective" cone resistance calculated as q_t - u_2.



Zone	Soil Behavior Type
1	Sensitive, fine grained
2	Organic soils-peats
3	Clays- clay to silty clay
4	Silt mixtures- clayey silt to silty clay
5	Sand mixtures- silty sand to sandy silt
6	Sands- clean sand to silty sand
7	Gravelly sand to sand
8	Very stiff sand to clayey sand
9	Very stiff, fine grained

Figure 6 Results from the Wellington Avenue site plotted on the classification chart proposed by Robertson (1990)



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1	Sensitive fine grained
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5	Sand mixtures- silty sand to sandy silt
6	Sands- clean sand to silty sand

Figure 7 Results from the Wellington Avenue site plotted on the classification chart proposed by Jefferies and Davies (1991)

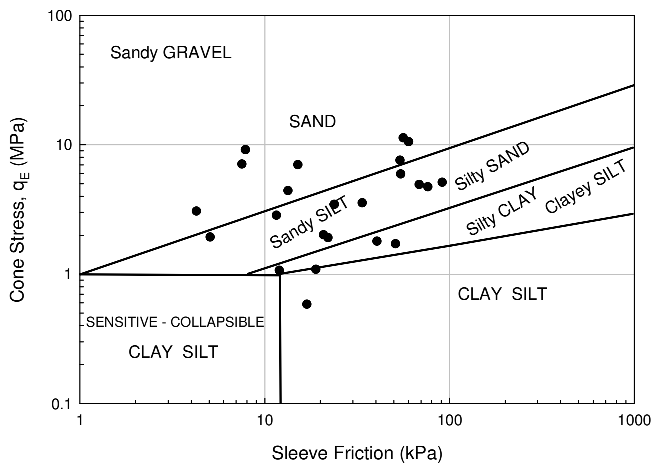

Figure 8 Results from the Wellington Avenue site plotted on the classification chart proposed by Fellenius and Eslami (2000)

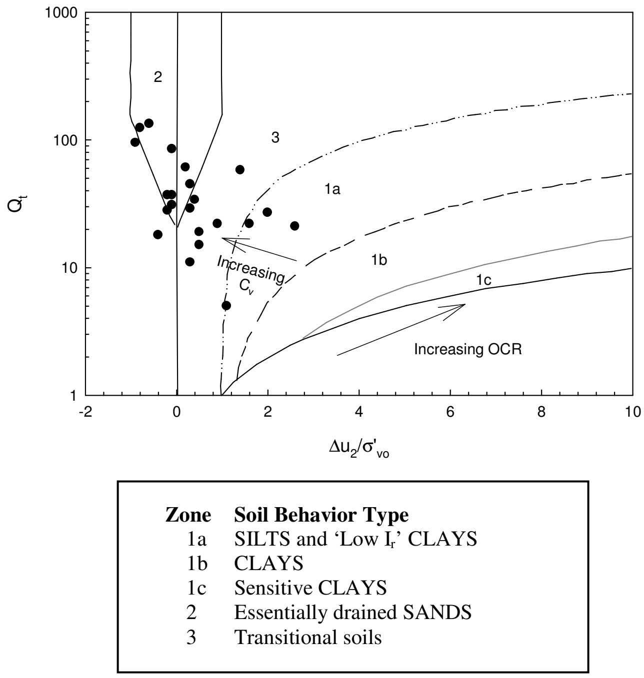

Figure 9 Results from the Wellington Avenue site plotted on the classification chart proposed by Schneider et al. (2008)

The CPT results indicate that the silt from this study plots over a wide range of soil zones from clays to sands. The first three charts (Figures 6 through 8) do not have a specific zone for silt and thus identification of uniform silt is difficult. However, Figure 9 does have a zone for "silt" in which 18% of the data plotted in this zone. However, in general the charts were ineffective in identifying the silt in this case.

The charts were effective in characterizing the engineering behavior of the silt in this study. In engineering practice it is common to treat non-plastic silts as sands. Therefore, identification of the silt as sand-like would provide a reasonable representation of its engineering behavior. Figures 6 through 8 identified the silt as "silt mixtures" or "sands" in 77% of the cases. The low excess pore water pressures generated during penetration is reflected in the chart in Figure 9 that classified the soil as "essentially drained sands", "transitional soils", and "silt". These soils would likely be considered sand-like in geotechnical analysis.

4.2 Friction Angle

Effective stress friction angle is used frequently in geotechnical analyses in non-plastic silt. With the exception of during pile driving, non-plastic silt is typically assumed to be drained over the time scales of construction. Peak effective stress friction angle was estimated from the CPT data at the Farmers' Market site using two correlations. The first correlation was developed for sands by Kulhawy and Mayne (1990) and defined by the following equation:

$$\phi' = \tan^{-1}\left[0.1 + 0.381\log\left(\frac{q_c}{\sigma_{vo}'}\right)\right] \tag{6}$$

where q_c=measured cone tip resistance (not corrected for unequal area effects). In sands, excess pore water pressures are minimal and thus q_c is approximately equal to q_t. However, since excess pore water pressures were generated in the silt, q_t was used instead of q_c in Equation 6. The ϕ' profile calculated using Equation 6 is shown in Figure 10.

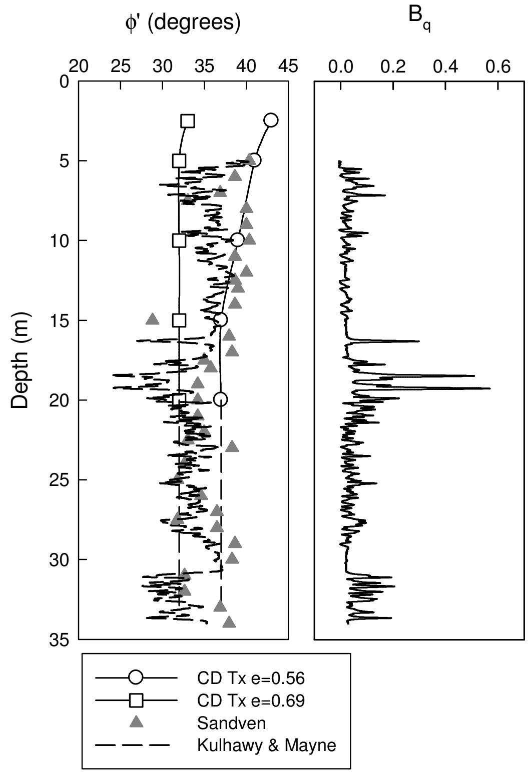

Figure 10 Typical profile of effective stress friction angle predicted with CPT correlations and upper and lower bounds measured in laboratory tests on Farmers' Market silt

Peak friction angle was also estimated from the CPT data using an approach summarized in Sandven (2003). The approach is based on bearing capacity theory that is used to perform an effective stress analysis from two normalized cone parameters: the pore pressure ratio (B_q) defined earlier by Equation 5c and the cone resistance number defined by the following equation:

$$N_m = \frac{q_t - \sigma_{v0}}{\sigma_{v0}' + a} \tag{7}$$

where a=attraction (=$c'\tan\phi'$). Equations are presented that describe the relationship between ϕ', N_m, B_q for a specified value of attraction and plastification angle (β). β describes the extent of plastified zones around the cone (Sandven 2003). In this study, the friction angle was determined at 1 m depth intervals, as shown in Figure 10, from the calculated values of N_m and B_q and the chart in Figure 11. Considering the silts are non-plastic (i.e. c'=0) the attraction was assumed to be zero.

The quality of the CPT friction angle correlations was assessed from the peak effective friction angles that were measured in triaxial compression tests on reconstituted samples of Farmers' Market silt. Figure 10 shows the measured friction angle representing the upper and lower bounds of void ratio anticipated at the site. The triaxial test data are plotted in the figure at the same initial vertical effective stresses estimated in the field. Note that the friction angles decrease with depth where they eventually become constant. The higher fiction angles at shallower depths are caused by curvature in the failure envelope due to dilation at low confining pressures. The friction angle decreases with depth as the soil becomes less dilative with increasing effective confining pressure. The denser silt (e=0.56) is more dilatant which explains why the friction angle becomes constant much deeper in the profile (15 meters depth) in comparison to the looser silt (e=0.69).

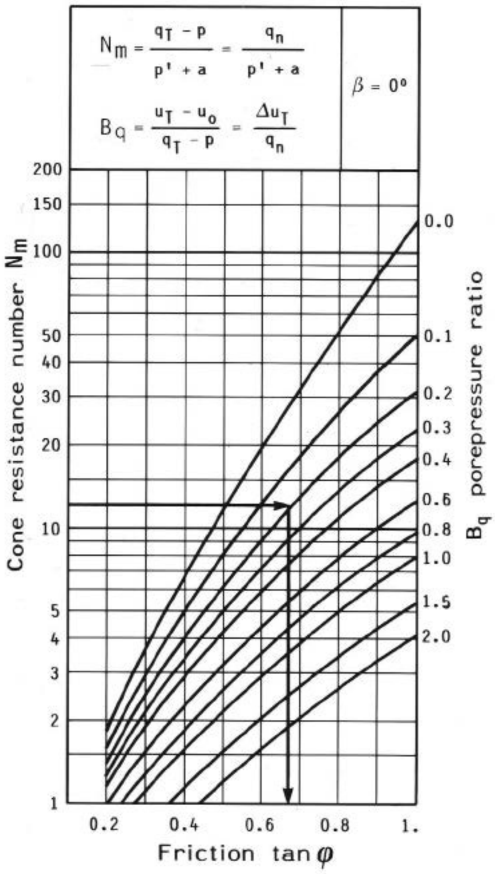

Figure 11 Chart used to estimate effective stress friction angle from CPT data in this study (Sandven 2003)

Comparing all results in Figure 10 the CPT predictions from Kulhawy and Mayne generally fall within the upper and lower bounds of friction angle with the exception of the depth intervals from about 16 to 20 m and 30 to 33 m where the low tip resistance predicted much lower friction angles (< 30 degrees). The lower tip resistance appears to correlate with high positive excess pore water pressures as shown by the B_q profile shown on the right side of Figure 10. Therefore, the inaccuracies in the prediction are likely due to the generation of pore pressures during penetration. However, the agreement is very good at other depth intervals where the excess pore water pressures are minimal.

Excess pore water pressures generated during penetration can skew the friction angle predicted by Kulhawy and Mayne causing underprediction of friction angle in contractive silt and overprediction in dilative silt. Therefore, it may be best only to apply the correlation in situations where there is minimal generation of excess pore pressures. Alternatively, the cone could be advanced at a much slower rate (even temporarily) to establish a drained condition.

The correlation by Sandven, which accounts for excess pore pressure generation during penetration, showed slightly higher friction angles than Kulhawy and Mayne. The predictions showed very good agreement with the laboratory data generally following the upper bound estimates of friction angle. Sandven did not underpredict friction angle in zones where high positive excess pore pressures were generated. This suggests that this approach is accurately accounting for the effects of pore pressure generation during penetration leading to more accurate estimates of effective stress friction angle.

4.3 Shear Wave Velocity

Shear wave velocity (V_s) is a dynamic soil property used in site response analysis and site classification for seismic design. Shear wave velocity is most influenced by effective confining pressure and void ratio but is also influenced by the age of the soil deposit, cementation, and stress history. Shear wave velocity (in m/s) was calculated using three different correlations including Hegazy and Mayne (1995), Mayne (2007), and Robertson (2009), respectively:

$$V_s = [(10.1\log_{10} q_t) - 11.4]^{1.67}[100 f_s / q_t]^{0.3} \quad (8)$$

$$V_s = 118.8\log_{10}(f_s) + 18.5 \quad (9)$$

$$V_s = [10^{(0.55 I_c + 1.68)}(q_t - \sigma_v)/P_a]^{0.5} \quad (10)$$

All stress parameters above are in units of kPa. The relationship proposed by Robertson (2009) is recommended for most Holocene to Pleistocene age deposits and is consistent with the age of the silt deposits in this study.

Shear wave velocity was measured in situ with the seismic cone at 1 m depth intervals. Typical profiles of shear wave velocity using the CPT correlations and seismic cone measurements are shown in Figure 12. It is clear from the figure that Robertson's correlation most closely matched the measured values. To better quantify the differences, Figure 13 plots histograms of the ratio of measured to predicted shear wave velocity for all three methods using all CPT data. It can be observed from this figure that Robertson's correlation was the most accurate with a mean (bias) of 0.93 and a standard deviation of 0.12. However, Mayne (2007) has comparable accuracy.

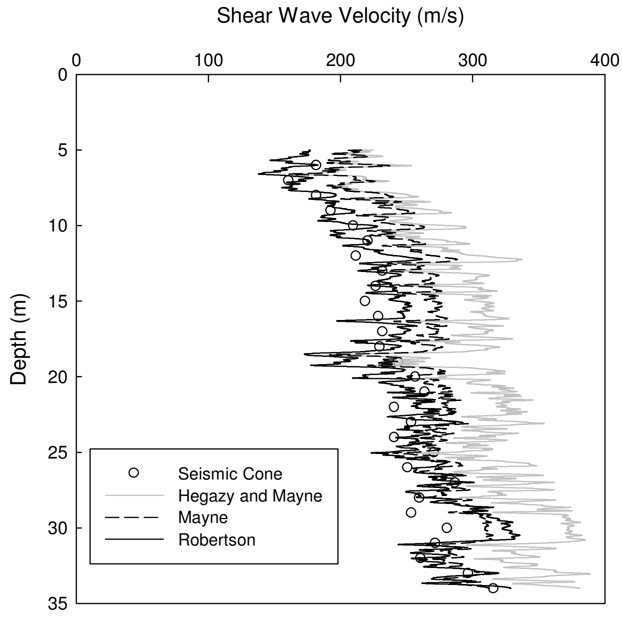

Figure 12 Typical profiles of shear wave velocity determined from CPT correlations and measured with the seismic cone

As shown in Figure 13 the bias ranged from 0.8 to 0.93 depending on the method causing an overprediction of shear wave velocity. The variability was similar among all three methods. The bias may fall within the uncertainty of the seismic cone measurement method itself. However, assuming that the seismic cone represents an accurate measurement, the calibration parameters in the correlations could be adjusted to remove the bias (i.e. mean=1.0).

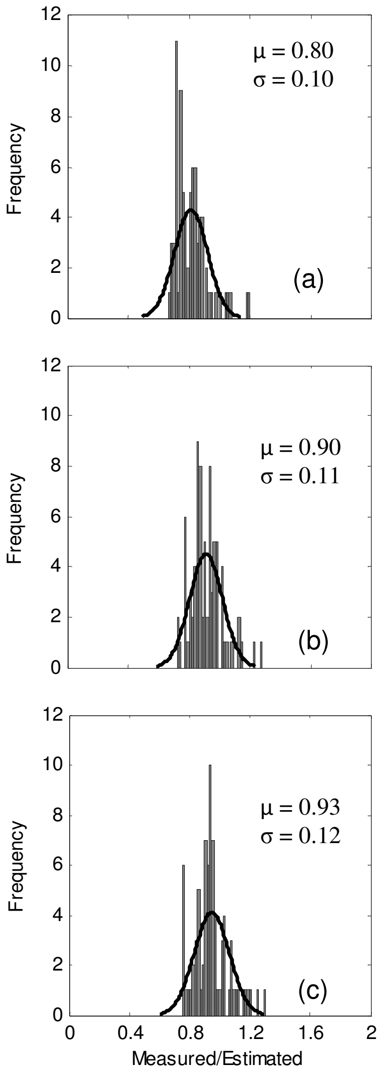

Figure 13 Histograms of shear wave velocity results for (a) Hegazy and Mayne 1995, (b) Mayne 2007, and (c) Robertson 2009

4.4 Cyclic Resistance Ratio

CRR is used in the assessment of seismic liquefaction potential. CPT-based CRR correlations from the literature are often based on case histories of liquefaction following major seismic events. Profiles of CRR for a magnitude 7.5 event were calculated for the Farmers' Market site using Robertson and Wride (1998) and Moss et al. (2006). Robertson and Wride represents the current standard of practice as outlined in Youd et al. (2001). Moss et al. expanded the liquefaction database and provided a probabilistic framework. Both methods have limited case history data from soils with non-plastic fines contents above 35% making their applicability in uniform silt, for example, uncertain. Similar comparisons have been made by the authors (Bradshaw et al. 2007c).

The calculation of CRR using Robertson and Wride (1998) is based on the calculation of a normalized cone tip resistance given by:

$$q_{c1N} = (P_a / \sigma'_{v0})^n (q_c / P_a) \quad (11)$$

where n=exponent typically ranging from 0.5 to 1.0. The reference pressure in Equation 11 must be in the same units as the overburden stress and the measured cone tip resistance. Again due to pore pressure generation in the silt, q_t was used in Equation 11 for q_c. The normalized cone tip resistance is then corrected to an equivalent clean sand normalized penetration resistance by:

$$(q_{c1N})_{cs} = K_c q_{c1N} \quad (12)$$

where K_c=correction factor depending on soil type. K_c is based on the calculation of the soil behavior index (I_c) given by:

$$I_c = \left[(3.47 - \log Q)^2 + (1.22 + \log F_r)^2\right]^{0.5} \quad (13)$$

An I_c of less than 1.64 is considered a clean sand and thus no correction is made (i.e. K_c=1.). An I_c greater than 1.64 indicates a sand containing fines and thus K_c is calculated from the following equation:

$$K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88 \quad (14)$$

An I_c greater than 2.6 indicates silt mixtures and clays. The average I_c value at the Farmers market site was 2.48 with a standard deviation of 0.22. Therefore, consistent with recommendations by Youd et al. (2001) for I_c<2.6 a value of 0.5 was used for n in Equation 11 and a value of 0.7 was used for I_c ≥2.6. The CRR was calculated at the Farmer's Market site using the following equations:

$$CRR = 0.833\left[(q_{c1N})_{cs}/1000\right] + 0.05 \quad ;(q_{c1N})_{cs} < 50 \quad (15)$$

$$CRR = 93\left[(q_{c1N})_{cs}/1000\right]^3 + 0.08 \quad ;50 \le (q_{c1N})_{cs} < 160 \quad (16)$$

The CRR profile calculated using Robertson and Wride (1998) is shown as a solid line in Figure 14. Note that the CRR profile shown in the figure is corrected for overburden stress using K_σ values recommended by Youd et al. (2001).

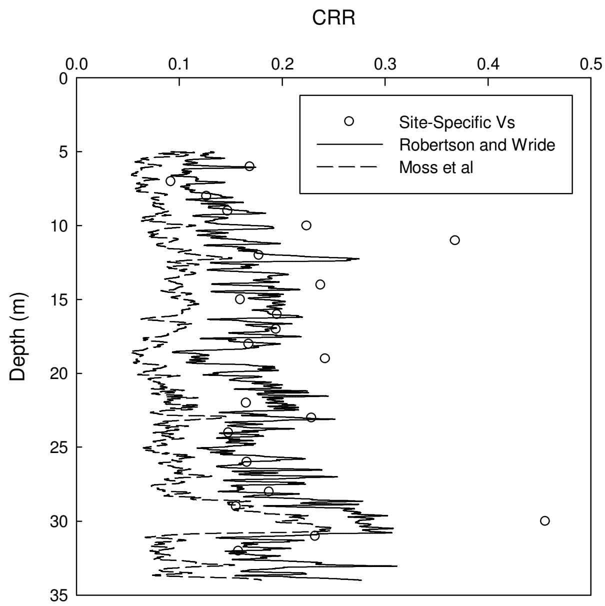

Figure 14 Typical CRR profiles determined from CPT correlations and from a soil-specific shear wave velocity correlation

The CRR correlation by Moss et al. (2006) is based on the calculation of an overburden stress corrected cone tip resistance:

$$q_{c1} = (P_a / \sigma'_{v0})^c q_c \quad (17)$$

where c=tip normalization exponent. The exponent c in Equation 17 is similar in function to the exponent n in Equation 11. However, c is determined by an iterative process as described in Moss et al. The average c exponent for the Farmers' Market was 0.41 with a standard deviation of 0.06. Again, q_t was used for q_c in Equation 17.

The CRR was calculated from the following equation (Moss et al. 2006):

$$CRR = \exp\left\{\begin{bmatrix} q_{c1}^{1.045} + 0.110R_f q_{c1} + 0.001R_f + c(1+0.85R_f) - 0.848\ln(M_w) \\ -0.002\ln(\sigma_v') - 20.923 + 1.632\Phi^{-1}(P_L) \end{bmatrix} / 7.177\right\} \quad (18)$$

where q_{c1}=normalized tip resistance (in MPa), R_f=friction ratio ($= f_s/q_c \times 100\%$), M_w=moment magnitude, σ_v'=effective vertical stress, $\Phi^{-1}(P_L)$=inverse cumulative distribution function. Moss et al. suggests using a P_L of 15% for a deterministic assessment of liquefaction potential that would be comparable to the correlation developed by Robertson and Wride. The profile of CRR using Moss et al. (2006) is shown as a dashed line in Figure 14.

A CRR profile was calculated using the soil-specific shear wave velocity correlation (Equation 3) and plotted as open circles in Figure 14. The CRR was not corrected for overburden stress as the soil-specific CRR-V_{s1} correlation was not influenced by overburden stress as discussed earlier. As shown in Figure 14, the CRR profile predicted by Robertson and Wride shows very good agreement with the V_s-based correlation. The CRR profile predicted by Moss et al. was approximately 35% lower on average.

The difference between the two CPT correlations is partly attributed to the differences in the way the soil type is treated. For example, Robertson and Wride adjusts the CRR by a factor based on the identified soil behavior index. Whereas Moss et al. accounts for soil behavior in the calculation of the cone tip normalization exponent. Moss et al. yielded a more conservative CRR profile but did not appear to be as accurate as Robertson and Wride based on the profile developed from the soil-specific shear wave velocity correlation.

5. CONCLUSIONS

This paper utilized CPT data from two silt sites in Rhode Island along with laboratory measurements in the same soils to assess existing CPT correlations for soil classification and estimation of effective stress friction angle, shear wave velocity, and cyclic resistance ratio. Based on the results presented in this paper the following conclusions are made:

- Cone penetration in the uniformly graded non-plastic silts in this study was primarily a partially drained to drained process. Therefore, the measured cone tip resistance was corrected for unequal area effects.

- All four of the CPT soil classification charts that were investigated (Jefferies and Davies 1991; Robertson 1990; Fellenius and Eslami 2000; Schneider et al. 2008) were ineffective in identifying uniform silt. However, they were effective in characterizing its engineering behavior by identifying the silt as sand-like rather than clay-like.

- The correlation for friction angle by Kulhawy and Mayne (1990) predicted friction angles that were within the laboratory-measured values in cases where excess pore water pressures were minimal. The correlation by Sandven (2003) provided better predictions of friction angle because it accounted for the effects of excess pore water pressure during cone penetration.

- The correlation for shear wave velocity by Robertson (2009) was the most accurate of the three correlations investigated with a bias of 0.93 based on a comparison to seismic cone measurements.

- The correlation for CRR by Robertson and Wride (1998) yielded results that were similar to the soil-specific shear wave velocity correlation developed for the study site. The CRR correlation by Moss et al. (2006) yielded results that were approximately 35% lower.

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